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Observations of Structural Damage Caused by Hurricane Katrina on the Mississippi Gulf Coast

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OBSERVATIONS OF STRUCTURAL DAMAGE CAUSED BY HURRICANE KATRINA ON THE MISSISSIPPI GULF COAST

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Abstract

The loads associated with Hurricane Katrina led to the destruction or severe damage of approximately 130,000 homes and over 200 deaths in the state of Mississippi. This paper discusses the results of a field inspection of structural damage along the state's Gulf Coast area caused by this hurricane. It was found that reinforced concrete, steel frame, and heavy timber structures generally performed well, with minimal structural damage. Precast concrete, light frame wood, and bridge structures generally performed poorly. Non-structural components of all building types, in particular facades and interior partitions subjected to storm surge, were typically destroyed. For various structures, the primary cause of failure was found to be insufficient connection strength. A comparison of Katrina's storm surge and wind loads is made to those specified in current design standards. It was found that Katrina's forces exceeded those specified in design standards in many parts of the state.

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Introduction

Hurricane Katrina was among the worst natural disasters in U.S. history in terms of geographical coverage, structural damage, and accompanying fatalities. Katrina first made landfall in south Florida on August 25, 2005 as a Category 1 hurricane, with wind speeds of approximately 36 m/s and gusts to 40 m/s. Atmospheric and ocean conditions were conducive to rapid intensification, which led to Katrina attaining major hurricane (Category 3) status on the afternoon of the 26th. This intensification was also accompanied by an unusual expansion outwards of hurricane-force winds, transforming the storm into a large hurricane typically only seen in the Pacific Ocean. Continuing to strengthen and move northwards during the next 48 hours, Katrina reached maximum wind speeds of 76 m/s (172 mph) (Category 5) on the morning of Sunday, August 28th. Katrina's hurricane-force winds extended 192 km from the storm center, and tropical storm-force winds 368 km outwards. As such, Katrina was significantly larger than Hurrican Camille, the benchmark used for Gulf of Mexico hurricanes hitting the Mississippi coast since August 17, 1969. Though Camille had peak wind speeds of 84 m/s, Katrina moved slower than Camille, thereby increasing the storm surge potential and time of wind exposure.

Katrina made landfall on the morning of August 29 in Buras, LA, with a central pressure of 923 mb, the 4th lowest on record in the U.S. for a landfalling Atlantic storm. The size of the hurricane caused unprecedented destruction, resulting in a record storm surge in southeast Louisiana, coastal Mississippi, and coastal Alabama, and a wide swath of damage for the same region but extending over 200 km inland in some regions. In Mississippi, about 68,000 homes were destroyed, and another 65,000 suffered major damage. As of this writing, the total death toll is between 1,300 and 1,400, with thousands more still unaccounted for. Between 200 and

250 deaths occurred among residents of the Mississippi coastal area. This places Katrina fifth in terms of hurricane fatalities, and the strom is the sixth deadliest natural disaster in U.S. history.

Several weeks after the hurricane struck, in the last week of October 2005, a team of researchers from Mississippi State University traveled along the coast on State Highway 90 from Biloxi, MS to Waveland, MS, to survey and document the structural damage (White et al. 2006.) This route represents approximately two-thirds of the Mississippi coast and includes some of the most highly-damaged areas. All cities along this route were surveyed, including Waveland, Pass Christian, Long Beach, Gulfport, and Biloxi (Fig. 1.) The purpose of this paper is to summarize these findings.

Hurricane Action

There are two primary types of loads that structures must content with when exposed to hurricanes. Water loads are created as a result of increases in water level and include the forces resulting from the movement of water onto land as an area becomes inundated. Wind loads develop sustained and dynamic conditions as a result of the hurricane's winds, as influenced by topography, ground cover, and adjacent obstructions.

Water Loads

In the initial stages of the hurricane, structures very near the coastline may be subjected to the impact of large surface waves. Much of this energy is absorbed as the waves break in shallow water approaching land, however. As time progresses, wind and wave action gradually raise the surface water level and propel the water further inland, resulting in a storm surge. A secondary contribution to surge is from the reduced barometric pressure within the storm, which causes a dome of water higher than the surrounding ocean. The surge rises gradually at first, then more rapidly as the storm makes landfall. Thus, the storm surge is relatively gradual and does not occur as a tidal wave, though the large wind-generated waves moving on top of the surging waters may create this impression.

The surge typically lasts several hours and affects about 160 km of coastline. Storm surge elevations typically vary from 1.5 to 7.5 m depending on a variety of hurricane conditions. At this stage, shoreline structures are subjected to a water head and associated lateral as well as buoyancy loads. The large waves associated with the raised water level during the storm surge may now cause significant structural damage. These waves no longer break offshore but travel over the land surface and may break upon coastline structures. This high wave action is perhaps the most severe structural load during the storm, and is the cause of most of the damage reported in this paper. Additional associated loads include hydrodynamic forces, water-borne debris impact, and foundation erosion.

Factors which affect storm surge elevation include: storm size; storm central pressure (lower interior atmospheric pressure increases the water level); maximum wind speed; bathymetry (as the surface currents driven by the wind reach shallow coastlines, bottom friction impedes the seaward return flow near the bottom, causing water to pile up; shallow areas with a gradual slope such as the Mississippi coastline will experience greater storm surges); speed of the system (as a slow moving hurricane has a longer time to transport water onshore, slow systems are associated with higher storm surge); wave setup (the super-elevation required to drive the underflow, which can be quite large in hurricane conditions); and track angle (storms which make landfall perpendicular to the coastline generally produce larger storm surges.) The strength of the surge in Mississippi was unexpected because the storm was slightly weaker than

Hurricane Camille in 1969. However, Camille came from the south-southeast direction, whereas Katrina slammed inland directly from the south along the Louisiana-Mississippi border, probably the worst possible track for Mississippi. Katrina also moved a little slower than Camille, allowing more time for the water to pile up. Although not directly a cause of the storm surge, Katrina's impact also began at high tide, which added approximately one foot to the surge.

<u>Wind Loads</u>

Wind is fundamentally driven by differences in pressure, and there are several types of high-force winds that accompany a hurricane. Hurricane intensity is defined by sustained winds, which is the average wind speed over a specified period of time at 10 m above the ground. In the Atlantic, this averaging is performed over a one-minute period. Wind gusts typically last for less than 20 seconds and are usually from 1.25 to 1.65 times larger than sustained winds, depending on topography. Note that ASCE 7 (2003) considers the average wind speed over three seconds a design wind gust. Some wind gusts are caused by downbursts, a strong downdraft that exits the base of a thunderstorm and spreads out at the earth's surface.

As hurricanes make landfall, interactions with the thunderstorms form columns of rapidly rotating air that may contact the ground and develop tornadoes. Officially 11 hurricane-related tornadoes were reported in Mississippi, though there were several dozen additional unofficial reports of tornado sightings. Finally, mesovortices may form in major hurricanes, which are whirling vortices that form at the boundary of the eyewall and eye where there is a tremendous change in wind speed. Mesovortices are often five to ten times wider than a tornado, with wind speeds up to 90 m/s. Little information is available on mesovortex formation in Katrina, but possible mesovortices were identified in the eyewall by satellite imaging.

High winds primarily damage roofs and exterior structural components, but generally pose much less of a threat to structures than storm surge and wave action, which can produce loads orders of magnitude higher, and are severe enough in many cases to destroy entire structures. An additional load associated with high wind is air-borne debris impact.

Damage Observations

Three general types of structures were surveyed: commercial buildings, residential (apartment buildings and single-family homes), and select pieces of the civil infrastructure (bridges). For this study, observations are grouped into construction type (bridges, reinforced concrete buildings, steel buildings, and wood buildings) rather than occupancy type. Unless otherwise noted, the damage described occurred to buildings located close to the coastline (generally within several hundred meters of the coast), and was due to storm surge.

<u>Bridges</u>

Biloxi Bay Bridge

Constructed as a 2.4 km, 4-lane prestressed concrete highway bridge, the Biloxi Bay Bridge was part of US 90 and connected Biloxi to Ocean Springs (Fig. 1.) All of the spans of the bridge's superstructure (i.e. deck and girders) were raised and pushed in a northeasterly direction, dropping the west side of the superstructure from the supporting pier (Fig. 2.) The piers below appear undamaged, and many of the spans are not damaged severely. The bridge girders were not constrained to the bearings (Fig. 3), and the surge simply lifted the spans from the supports. Thus it appears that connection inadequacy was the cause of failure. A significant contributing factor may be the buoyant force. Given that submerged concrete has its self-weight effectively reduced by over 40%, as well as the possibility of air being trapped under the deck between adjacent girders as the surge rises, the failure is not surprising. The surge also caused significant scour beneath the road on top of the abutments as well as the abutments themselves (Fig. 4.)

Bay St. Louis Bridge

This was a 3.2 km prestressed concrete bridge that spanned from Pass Christian to Bay St. Louis (Fig. 1) This bridge lost all of its spans as they were pushed completely off of the pier supports (Fig. 5.) There was some pier damage as well. As all spans separated from the piers at the bearings and the visible spans appear intact, inadequate bearing up-lift strength was the cause of failure, as with the Biloxi Bridge. In this case, the bearing was apparently tied to the superstructure but the strength of the connection as well as its embedment into the pier was simply inadequate (Fig. 6.) Serious scour was also observed (Fig. 7.)

<u>Reinforced Concrete Structures</u>

Nearly all of the reinforced concrete (RC) structural frames that were identified appeared to have performed well, with no apparent displacement, damage, or visible cracks. However, this was not the case for building façades or interior walls, as these were often significantly damaged or entirely missing from the structure if it were struck by the storm surge. Figure 8 presents a typical structurally-undamaged low-rise RC frame. Figure 9 illustrates another common RC structure on the Mississippi coast, where a slab and column system is used to support and elevate a wood structure above. In this instance, the wood structure was completely destroyed. A private residence composed of an RC frame as well as RC exterior walls sustained no apparent damage to the frame or exterior walls (Fig. 10.)

On some buildings, the destruction of the façade and interior partitions may have alleviated more severe damage by preventing an extreme storm surge load to be transferred to the structural frame. Similar to the smaller-scale structures, high-rise (10+ story) RC buildings on the coast often had the contents of the first one or two floors removed by the surge, but the upper floors as well as the entire structural frame appeared to have sustained no damage. Figure 11 represents a typical structure of this type. It may be expected that the larger RC buildings would survive as compared to smaller structures, as lower-floor member capacities are clearly greater than those composing 1-3 story buildings. Increased building mass may also serve to resist displacement due to wave impact as well as lateral-pressure induced sliding. However, based on the observed failures of other types of structures, the authors contend that it is not primarily member capacity nor building mass but rather the connection strength of RC that lead to the survival of these structures.

One of the two observed RC failures was the collapse of columns holding what may have been a pergola roof. These columns failed at the base, a failure which may have been mitigated if stirrups were placed in the columns to provide confinement (Fig. 12.) The other observed damaged RC building (Fig. 13) appears to have been struck by a large casino barge that washed onto the shore (not shown in the Figure), where only the impacted corner was damaged but the remainder of the structure was unaffected. The local damage appears to have been well-contained and did not visibly propagate to additional structural members. Although no additional RC failures were identified in this study, others have reported a small number of RC structural failures (Roberson et al. 2006). These include flat and pre-stressed concrete building slabs, which may have failed due to moment reversals caused by the storm surge uplift.

Precast Concrete Structures

Unlike the RC structures, many of the precast concrete (PC) buildings observed sustained significant structural damage. The members themselves appear to have had sufficient capacity, however, as all observed failures occurred at the connections. Figure 14 shows an RC frame upon which PC floor slabs were placed. Although the RC frame was undamaged, the PC slabs appeared to have detached at the connections and slid to the northwest. Figure 15 shows a collapsed structure composed of PC girders supported by RC columns. This structure appears to have been used to support a wood superstructure. Here again the PC members were undamaged but failures occurred at the connections. A detail of the failed connection is shown in Figures 16 and 17. A PC pedestrian bridge is also shown to have failed at the connections (Fig. 18.) Here both girders were detached from the supporting column, and the canopy detached from the girders (which served as guardrails), with the steel connectors still visible on the top of the girders. The second floor of a PC parking structure failed (Fig. 19) when the deck T-sections were pulled from their supports on the spandrel beam. Some spandrel beams supporting the second floor also collapsed. As with the observed bridge failures, buoyancy likely was a significant factor. Here again the primary cause of failure appeared to be a lack of sufficient connection strength rather than member capacity. It should be noted that other damage surveys have identified failed PC members in parking garages, apparently due to a moment reversal caused by the surge uplift force, as occurred with the RC slab failures identified (Roberson et al. 2006). In these cases, increasing connection strength alone may not have prevented the failures.

Steel Frame Structures

Similar to the RC structures, most steel frames appeared to have survived intact, with little or no damage. As with the RC structures, this does not include the façade and interior walls, whether made of masonry, wood, or steel studs. Figures 20-23 show typical such structures. There were some exceptions, however. Figure 24 shows a collapsed steel frame. Here the metal roof as well as the supporting purlins were bent upward, perhaps by wind, resulting in a loss of lateral stability that caused the first frame to collapse inward. Open web steel joist construction did not perform as well as wide-flange or built-up steel frame construction, a structural type for which several failures were observed. Figure 25 illustrates such a collapse. Here it appears that the far wall was toppled by the surge, causing the roof to collapse. Based on failures similar to those shown in Figure 24, as well as the observation that most steel frame failures occurred for structures with relatively slender members, it appears that the primary area of concern for the performance of steel structures under hurricane loads is stiffness and lateral stability rather than insufficient member capacity.

Wood Structures

As a rule, light-frame wood structures along the coastline were almost entirely destroyed. The typical remains of coastline residence is shown in Fig. 26, where sub-structural columns (often made of heavy-timber or RC) survived but the supported house did not. Unfortunately, large numbers of images such as this line the Mississippi coast. Primary failures were at the nailed connections, as searching through debris piles revealed that most of the wood members were intact. For wood structures that were not destroyed by storm surge, several common types of damage were seen:

1. Roof failures. This generally occurred away from the coastline where the water surge was not great enough to topple the structure. As expected, roof damage typically occurred near the edges rather than central portion of the roof, where uplift forces are highest. The sheathing panels appeared to have lifted whole from the roof, indicating a lack of connection strength rather than panel bending capacity. Figure 27 presents a commonly-observed type of roof sheathing failure. This observation was experimentally verified as well (Schiff et al., 1996). Higher winds produced more extensive damage to the roof structure itself, which was also frequently observed (Fig. 28.)

2. Siding and wall sheathing failures. As noted previously as being common on steel frame and RC frame buildings as well, siding and sheathing stripped from the structure is primarily a sign of wind damage (Fig. 29), though more extensive damage to sheathing may indicate high water loads. Again, most losses of siding are indicative of insufficient fastener strength.

3. Side-sway failures. Figure 30 is representative of this type of failure. Of the three general types of light-frame wood structure failures observed, these were much less common than the other two outlined above. The loss of lateral stability could have been induced either by a wind or storm surge overload.

Unlike light-frame construction, heavy timber and glued-laminated frame structures appeared to have fared as well as steel and RC. Of course the connection strength (typically bolted) and member stiffness are much greater than those associated with the dimensional lumber in light-frame construction. Common timber structural systems that survived were post-and-beam pier substructures used to support a light-frame wood house above (Fig. 31.) Other structures included docks (Fig. 32) and glued-laminated frames (Fig. 33.) Although damage was observed on some of these systems, in general it appeared to be minimal.

Comparison to Current Standards

ASCE 7, Minimum Design Loads for Buildings and Other Structures (ASCE 2002), provides design loads for wind and storm surge loads, and ASCE 24, Flood Resistant Design and Construction (ASCE 2002), provides additional guidance to mitigate storm surge. Most building codes, such as the International Building Code (2003) and International Residential Code (2003) incorporate ASCE 7 and 24 standards directly or by reference (note as of this writing, ASCE 7, ASCE 24, the IBC and IRC are in the process of being revised to interface more closely and the most current editions should be available in early 2006).

Storm Surge

Observations and data on Katrina's storm surge cycle generally do not exist because all of the tide gauges failed along the Mississippi coast as a result of the storm. Storm surge heights are thus estimated from computational simulation and post-storm high-water mark measurements. High water mark surveys were conducted by a variety of agencies, including the National Weather Service, the Army Corps of Engineers, the US Geological Survey, and private companies. Surge values between 8.5 and 9.4 m have been documented between Pearlington and Bay St. Louis, MS. High water marks between 6 and 8.2 m occurred between Bay St. Louis and Biloxi (Fig 1.) Ocean Springs and Pascagoula experienced smaller but still significant surge values ranging from 3.6 to 5.8 m (White et al. 2006.)

Data from simulations predicting water levels are producing similar values. The results of numerical simulation of the U.S. Army Corps of Engineers Advanced Circulation (ADCIRC) fully nonlinear hydrodynamic model (Luettich and Westerink 2000) are compared to observed water marks in Table 1. The table also presents Federal Emergency Management Agency (FEMA) Base Flood Elevations (BFE) found on Flood Insurance Rate Maps (FIRM) to the observed and simulated high-water levels for four communities along the Mississippi Gulf Coast. The BFE represents the water level associated with a flood that has a 1% probability of occurrence each year. In typical cases, ASCE 7 and ASCE 24 reference the FEMA BFE to the Design Flood Elevation (DFE). In each case considered in Table 1, the BFE was exceeded (typical along the coastline). Coastal high water elevations along the Mississippi Gulf Coast in most areas varied from Waveland to Gulfport from approximately 7.5-9 m, and from Gulfport to Pascagoula from about 4.5-6 m. In contrast, FEMA BFEs along the coast from Waveland to Gulfport varied from 3.3-5.8 m, while from Gulfport to Pascagoula ranged from 2.7-6 m (White et al. 2006.)

The estimated difference in storm surge design loads caused by the observed storm surge levels in Table 1 and the FEMA BFEs are shown in Table 2. Design loads (unfactored) are computed for a typical single-story house with a peak roof height of 4.5 m, using the simplified procedure outlined in ASCE 7 Section 5. Here surge load is taken as the sum of a hydrostatic component (approximately 15% of total load), a hydrodynamic component (5%), and wave impact (80%). The procedure outlined in ASCE 7 assumes a stillwater depth of 65% of the BFE and a wave height of 78%. In the sample calculations, water velocity is taken as 3 m/s (upper limit for the simplified design procedure), coefficient of drag is taken as 1.25 (minimum allowed), and the building is assumed to be in importance category 2 (typical for a residence), with dynamic pressure coefficient equal to 2.8. The building is assumed to be enclosed. Results in Table 2 are given in terms of force per unit length on the exterior building wall facing the surge. As seen in Table 2, estimated Katrina storm surge levels result in significantly larger forces (from 1.4 to 4.0 times) than those based on the pre-storm BFEs. The large difference is primarily due to the wave load, which is a function of the square of water depth.

Wind Gusts

Figure 34 shows superimposed ASCE 7 basic design wind speed gusts to wind gusts estimated for Hurricane Katrina. The Katrina wind gusts were estimated by numerical simulation from the National Oceanic and Atmospheric Administration (NOAA) Atlantic Oceanography and Meterology Lab H*Winds models, as well as in situ data collection (White et al. 2006.) Wind data collected from weather stations generally revealed lower gust speeds than the NOAA estimate, as shown in Table 3. The estimates are within reasonable agreement, however. As shown in Figure 34, the design winds are exceeded in a relatively narrow swath of land in the southeastern portion of the state. The maximum difference between the estimated Katrina wind speeds and the design wind speed is approximately 13 m/s, and occurs just south of the 49 m/s wind design speed contour, where the peak of the estimated 63 m/s wind gust contour appears. Based on the example residence above, assuming a 20 degree (approximately 4:12 pitch) gable roof in typical exposure C topography with importance factor 1.0, design wind pressures (unfactored) are given in Table 4 for a 49 m/s and 63 m/s wind. Values are computed using Method I in ASCE 7, a simplified analytical procedure. Considering both the main wind resisting structural system as well as components and cladding, the Katrina-estimated 63 m/s wind is expected to apply lateral pressures of approximately 1.6 times those of the 49 m/s design wind.

Conclusions and Recommendations

The performance of a variety of building types along the Mississippi coast exposed to the extreme loads imposed by Hurricane Katrina was summarized in this study. Based on these observations, several recommendations can be made.

1. Explore current land use policy. As most structural damage was caused by storm surge, the concentration should be on water loads rather than wind loads. Several possibilities should be considered with regard to land use and water loads, including restricting certain construction or building occupancy types, or increasing construction standards in certain areas. Many of these decisions have been already made, or are currently under consideration, by local authorities.

2. Reconsider design loads. Two issues are important: rate of return and the expected load. Considering the rate of return, a 100-year (i.e. 1% chance of a flood of this magnitude per year) flood is the typical rate of return used for flood design, while a 50-year wind is the basis for wind gust consideration. A determination must be made whether this rate of return adequate, or to consider a longer (or shorter) period of time. Considering the expected load associated with the rate of return, it would also be prudent to investigate whether the current hurricane design loads sufficiently reflect the actual imposed loads. A re-consideration of the appropriate design flood elevation should be made, as many coastal base flood elevations were significantly exceeded. As

of this writing, FEMA is in the process of updating BFEs. Similar to water loads, wind speeds were significantly exceeded in many parts of the state as well.

3. Maintain consistency in safety level. From a structural safety point of view, it makes little sense for structures of the same importance to be designed to different levels of reliability. The goal of a design standard is not only to ensure a minimum level of safety, but also to ensure consistency in reliability, the primary issue that Load and Resistance Factor Design standards attempt to address. This fundamental goal is difficult if not impossible to achieve if consistent standards are not enforced. Unlike its neighboring states, Mississippi currently has no statewide design standard.

4. Address specific hurricane design and construction standards for the various building types. It appears that most structural frames of steel, reinforced concrete, heavy timber, and glued-laminated wood performed well during the storm surge. With the possible exception of a lateral stability concern for steel frames and uplift loads which may cause negative moments for some concrete structures, this suggests that existing design and construction practices for these types of structures may be adequate. It is advisable to verify this adequacy under the expected storm surge loads, however, with particular regard to assessment of safety level. Conversely, precast concrete structures exposed to the storm surge load did not perform well. Member capacity generally appeared adequate but connection strength was often insufficient. Thus, it is recommended to investigate the adequacy of connection design strength.

Observations made for this study also suggest that bridge superstructures had adequate capacity. As with precast concrete buildings, a lack of connection restraint caused the observed failures. An investigation of bearing connection strength is recommended. Most light-frame wood structures subjected to storm surge were destroyed. It appears that most failures occurred at the fasteners. However, it is not known whether strengthened connections alone would have resulted in substantial decreases in damage, as overall structural system resistance to a lateral load such storm surge is much less than that found in the survived commercial-grade frames. An investigation to determine the desired and current design strengths is recommended, and how the gap between these can be closed. Particular attention should be paid to fastener strength.

Finally, facades of all types in general did not perform well. Here two design possibilities exist: facades are either designed to withstand storm surge or they are designed to break away to avoid overloading the structure. The latter case may be most appropriate for the lower floors of most coastline structures. Once a desired level of performance is determined, numerous ways exist to achieve this level with a variety of façade types.

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Table 1. Comparison of Base Flood Elevations and Katrina High Water Levels

MS Location	FEMA BFE (m)	Observed Katrina Water Level (m)	Simulated Katrina Water Level (m)
Pass Christian	3.9-5.5	7.6	7.6-9.1
Bay St. Louis	3.9-5.2	8.2	7.6-9.1
Gulfport	5.5	6.7	7.6-9.1
Pascagoula	3.0-3.9	3.6-5.2	4.5-6.1

Table 2. Comparison of BFE and Estimated Actual Surge Loads

MS Location	BFE Load (kN/m)	Estimated Actual Load (kN/m)	Actual / BFE
Pass Christian	854	1591	3.5 - 1.9
Bay St. Louis	767	1825	4.0 - 2.4
Gulfport	458	1245	2.7
Pascagoula	458	767	1.4 - 1.7

Table 3. Estimated and Measured Wind Gusts

MS Location	Measured Gust (m/s)	Simulated Gust (m/s)
Waveland	54-58	64
Biloxi	49-51	44
Ocean Springs	47	44
Pascagoula	44	42

Table 4. Comparison of Design and Measured Wind Gust Pressures

Lateral Pressure on Main Wind Resisting Structural System, Walls

	Wind Pressure		
Location on Wall*	49 m/s Wind	63 m/s Wind	Ratio, (49) / (63)
Interior (region C)	1.03 (kN/m^2)	1.66 (kN/m^2)	1.61
Edges (region A)	1.54	2.49	1.62

Uplift Pressure on Components and Cladding, Roof

	Wind Pressure		
Location on Roof*	49 m/s Wind	63 m/s Wind	Ratio, (49) / (63)
Interior (zone 1)	-1.13 (kN/m^2)	-1.82 (kN/m^2)	1.61
Edges (zone 2)	-1.90	-3.00	1.58
Corners (zone 3)	-2.78	-4.50	1.62

*Specific locations for ”regions” and “zones” are defined in ASCE 7, Figures 6.2 and 6.3.

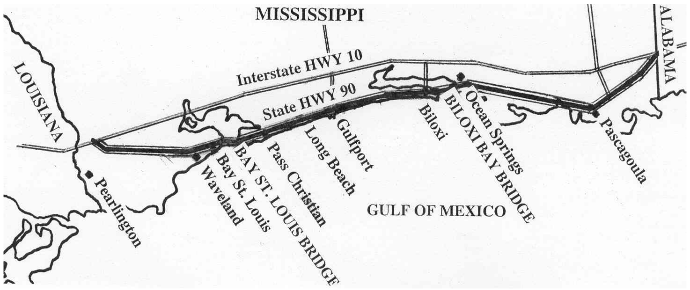

Figure 1. Extent of survey along State Highway 90.


Figure 2. Collapsed Spans of Biloxi Bay Bridge.


Figure 3. Biloxi Bay Bridge bearings show lack of adequate connection to superstructure.


Figure 4. Biloxi Bay Bridge road undermining.


Figure 5. Completely displaced spans of Bay St. Louis Bridge.


Figure 6. Insufficient connection strength of Bay St. Louis Bridge bearings.


Figure 7. Bay St. Louis Bridge abutment undermining.

Figure 8. Typical undamaged RC frame with destroyed façade and gutted interior.

Figure 9. Underside of a typical undamaged RC frame and slab, used to support a (destroyed) wood building above.


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Figure 11. Typical undamaged structure of high-rise RC frame, with first floor façade and contents destroyed.

Figure 12. RC failed columns.


Figure 13. RC frame building struck by casino boat.


Figure 14. Failed connections of PC slabs supported by a RC frame.


Figure 15. Collapsed PC girder system.


Figure 16. Failed PC column connection.


Figure 17. Failed PC girder connection.

Figure 18. Failed PC pedestrian bridge. Note detachment at connections.

Figure 19. Collapsed T-beams of PC parking structure.

Figure 20. Typical survived steel frame, with façade and interior contents removed by storm surge.

Figure 21. Typical survived steel frame.

Figure 22. Typical multi-story survived steel frame.

Figure 23. Survived steel frame.

Figure 24. Steel frame lateral stability failure.

Figure 25. Failed steel frame with open-web joist roof.

Figure 26. Typical remains of wood structure, where only pier system survived.

Figure 27. Typical wood roof sheathing failure. Note panel is removed whole and at roof edge.

Figure 28. Typical wood roof structural system failure.

Figure 29. Typical wood roof siding and sheathing failure.


Figure 30. Wood house side-sway failure.


Figure 31. Typical timber beam and pier system.


Figure 32. Survived glued-laminated dock.

Figure 33. Survived glued-laminated frame.

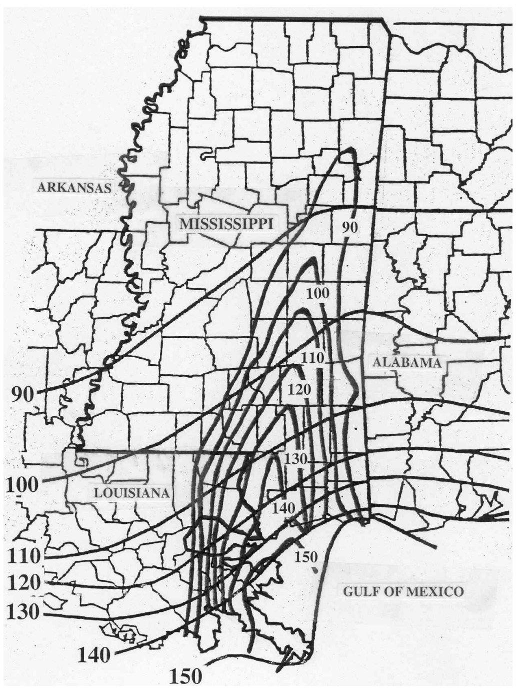

Figure 34. Code (horizontal contours) and Katrina (vertical contours) wind gusts (MPH).